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THE DESIGN OF A REINFORCED CONCRETE RAILWAY ARCH

by

HENRY WESLEY CLARK

A THESIS SUBMITTED FOR THE DEGREE OF
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INTRODUCTION

The design of a reinforced concrete railway arch was undertaken by the writer, as a thesis problem.

The general form of tables and sequence of operations in design were obtained from a set of notes furnished by W. S. Kinne, Associate Professor of Structural Engineering, University of Wisconsin.

Information, methods of procedure, and formulae were obtained from various books, the principal ones used being:

Modern Framed Structures, Part I..................
............by Johnson, Bryan and Turneaure
Principles of Reinforced Concret Construction.....
....................by Turneaure and Maurer
Reinforced Concrete Construction...................by Hool
Concrete, Plain and Reinforced......by Taylor and Thompson

THE DESIGN OF A REINFORCED CONCRETE RAILWAY ARCH.

Span 64'-0"   Rise 16'-0"   Ratio of rise to span .25

GENERAL DATA

Rise 16'-0"
Span 64'-0"
Depth of earth fill over crown 2'-0"
Depth of ballast above earth filling to base of rail 1'-6"
Live load Coopers' E 50

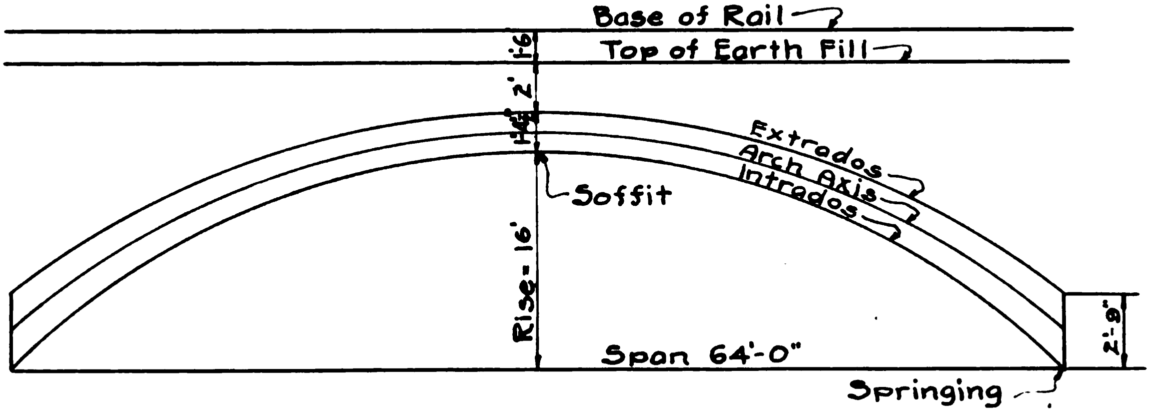

FIG. 1.

LOADINGS

Dead load. Actual weight of materials.

Earth fill (sand)	120 lbs per cu. ft.
Concrete, including reinforcement	150 lbs per cu. ft.
Ballast, ties and rails	200 lbs per lin. ft.

Live load.

A uniform load equivalent to Coopers' E 50.

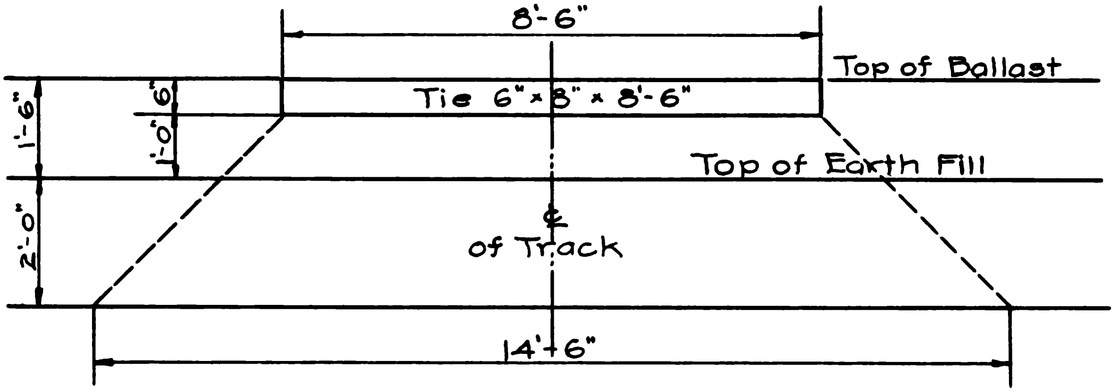

FIG. 2.

The equivalent uniform load is that load which will produce the same bending moment as the concentrated loads of Coopers' E 50, in a beam whose length equals the length of span loaded.

The load is assumed to be distributed over a width of 14'-6" as shown above.

Let w = equivalent uniform load per lineal foot per foot width in pounds.
m = moment for a single track in a beam of length l, in foot pounds.
l = length of span in feet.

$$w = \frac{8m}{14.5\,l^2}$$

From table on page 245 of Part I, Modern Framed Structures,

m for one rail = 1,819,400 foot pounds for a beam of 64'-0" span.

$$w = \frac{8(1,819,400)2}{14.5 \times 64 \times 64} = 490 \text{ lbs per sq. ft.}$$

For 1/2 span or 32'-0"

$$w = \frac{8(569,300)2}{14.5 \times 32 \times 32} = 610 \text{ lbs. per sq. ft.}$$

CONDITIONS OF CALCULATIONS.

E_s = 30,000,000 lbs. per sq. inch.
E_c = 2,000,000 lbs. per sq. inch.
n = 15
f_c = 500 lbs. per sq. inch exclusive of temperature stresses.
f_c = 750 lbs. per sq. inch including stress due to 20° F. variation.
f_s = 15,000 lbs. per sq. inch including stress due to 20° F. variation.
Tension on concrete to be neglected.
Maximum shear allowed on concrete, 75 lbs. per sq. inch.

DETERMINATION OF THE THICKNESS OF THE ARCH RING.

The thickness of the arch ring of the crown was determined from the following empirical formulae:

F. F. Weld Formula.

$$d = \sqrt{S} + 0.1S + 0.005w + 0.0025p$$

where d = depth of crown in inches
s = clear span in feet
w = live load lbs per sq. ft. uniformly distributed
p = weight of dead load above crown in pounds per sq. ft.

S = 64'-0"
w = 490 pounds
p = Assume 18'

$$p = \left(150 \times \frac{18}{12}\right) + 2(120) + 200 = 665 \text{ lbs.}$$

$$d = \sqrt{64} + 0.1(64) + 0.005(490) + 0.0025(665)$$
$$d = 8 + 6.4 + 2.45 + 1.66 = 18.51"$$

D. B. Luten Formula:

$$d = 4 + \frac{3S^2(r + 3F)}{4000r - S^2} + \left[\frac{L_t S^2}{30000r} \text{ or } \frac{L_m(S + 5r)}{150r}\right]$$

S = 64'-0"
r = 16'-0"
F = 3.5
L_t = 490
L_m = 128.75

$$d = 4 + \frac{3 \times 64^2 (16 + 3[3.5])}{4000(16) - 64^2} + \left[\frac{490 \times 64^2}{30000(16)} \text{ or } \frac{128.75(64 + 5 \cdot 16)}{150 \times 16}\right]$$

$$d = 4 + 5.43 + [4.18 \text{ or } 7.71]$$

$$d = 17.14"$$

Balet Formula for an arch of less than 1/3 the span.

$$d = 3.333n + 0.001\frac{WS}{r_a f_c}$$

reduce 15%
S = 64'-0"
$r_a = 16' - 1.5 \div 2 = 15.25'$
$f_c = 250 = \frac{1}{2}$ of 500

VALUE OF W. $W = (f + d + kr)\frac{S}{2} \times 130$

W - weight in pounds of 1/2 the arch
f - fill over crown in feet . . . .- 3.5'
d - assumed depth of arch ring in feet . . . . .- 1.5'
r = rise feet . . . .-16
s = span feet . . . .-64
k = factor from table . . . .- .255

$$W = (3.5 + 1.5 + .225 \times 16)\frac{64}{2} \times 130$$

$$W = 9.1 \times 32 \times 130 = 37{,}800 \text{ lbs.}$$

$$d = .85\left(\overline{3.333 \times .2} + .001\,\frac{37{,}800 \times 64}{250 \times 15.25}\right) = 1.11' = 13.3''$$

$$d_{crown} = \frac{18.51 + 17.14 + 13.3}{3} = \frac{48.95}{3} = 16.32''$$

$$\text{depth} = 16\tfrac{1}{2}'' \text{ at crown}$$

Vertical depth springing $= 2 \times 16.5 = 33''$

The depth of arch ring at crown was taken as the average value of the above determinations.

The depth of springing was made twice the crown depth.

DETERMINATION OF THE CURVATURE OF THE ARCH AXIS.

A trial arch was assumed in which the curves were taken as segments of circles.

The depth at crown was taken as 16½" and at springing as 33", from the previous determinations.

The radii of curves were found by means of the following formula, the chord and mid-ordinate being respectively the span and rise.

$$\text{Radius} = \frac{(\tfrac{1}{2}\text{ CHORD})^2 + (\text{MID-ORDINATE})^2}{2 \times \text{MID-ORDINATE}}$$

DETERMINATION OF DEAD LOADS AND THEIR ACTION LINES.

The horizontal distance between springing lines was divided into 20 equal divisions, ten on each side of the crown. Vertical lines marking these divisions were drawn from the intrados to the top of the ballast. The actual load on the arch was then reduced to an equivalent concrete load of 150 lbs. per cubic foot and the reduced load contour plotted as shown on Plate One. The values were taken from Table 1-a (see Page 5).

TABLE 1-A
COMPUTATION OF REDUCED LOAD CONTOUR AND DEAD LOADS

Horizontal Distance from Crown	Depth of Sand	Depth of Arch Ring	Reduced Depth of Sand	Reduced Depth of Ballast	Total Equ. Depth of Concrete	Average	Value of Dead Loads
0	2.0	1.375	1.60	1.33	4.305	4.35	2090
3.2	2.1	1.38	1.68	1.33	4.39	4.56	2190
6.4	2.5	1.40	2.0	1.33	4.73	5.00	2400
9.6	3.12	1.45	2.50	1.33	5.28	5.65	2710
12.8	4.00	1.50	3.20	1.33	6.03	6.56	3150
16.0	5.20	1.60	4.16	1.33	7.09	7.71	3700
19.2	6.65	1.70	5.31	1.33	8.34	9.11	4370
22.4	8.45	1.80	6.76	1.33	9.89	10.89	5230
25.6	10.70	2.00	8.56	1.33	11.89	13.13	6300
28.8	13.40	2.35	10.70	1.33	14.38	15.90	7640
32.0	16.70	2.75	13.35	1.33	17.43		37,780

TABLE 1-B
SECOND TRIAL - RIGHT HALF

0	2.0	1.375	1.60	1.33	4.305	4.35	2090
3.2	2.1	1.39	1.68	1.33	4.40	4.53	2175
6.4	2.4	1.41	1.92	1.33	4.66	4.88	2450
9.6	2.9	1.45	2.32	1.33	5.10	5.41	2600
12.8	3.6	1.52	2.88	1.33	5.73	6.17	2960
16.0	4.6	1.60	3.68	1.33	6.61	7.13	3420
19.2	5.75	1.72	4.60	1.33	7.65	8.29	3980
22.4	7.2	1.85	5.75	1.33	8.93	9.88	4740
25.6	8.95	2.35	7.15	1.33	10.83	12.20	5850
28.8	11.00	3.45	8.80	1.33	13.58	15.78	7560
32.0	13.45	5.90	10.75	1.33	17.98		37,825

The equivalent depth of concrete,- the vertical depth of arch ring $+ \frac{120}{150}$ x depth of earth filling $+ \frac{200}{150}$ (ballast), was laid out from the intrados and the Reduced Load contour drawn through the upper limits of the distances so laid out. All measurements were made in feet.

The amount of the dead load for each division was then determined. The volume between two longitudinal planes, one foot apart, and two vertical planes, perpendicular to span length, passed through the vertical division lines, included between the reduced load contour and the intrados, multiplied by 150, gave the amount of the dead load for each division.

The action line of each load was taken through the center of the section.

The next step was to pass an equilibrium polygon through the centers of the arch axis at the springing line and the crown, by a graphical method. This was done in the following order: (See Plate I, left half of arch)

1. The lead line m-n was laid off first to a scale of 1" = 6,000 lbs.
2. A trial pole was taken on the horizontal from n the point representing the end of the load nearest the crown.
3. Rays were then drawn from the pole to the load points as shown and the corresponding string polygon drawn between the load lines. The first and last rays were drawn to an intersection, which intersection is on the vertical containing the center of gravity of all the loads on that half of the arch.
4. In any arch the horizontal ray through the center of the arch ring and the ray through the center of ring at the springing line will intersect at the vertical line drawn through the center of gravity of the loads on the half of the arch. The horizontal was then drawn through the arch axis at the crown to the intersection point with the vertical containing the center of gravity of the loads. From this point a line was drawn to the point at the springing line on the arch axis. Then a line from m parallel to the line just found intersects the horizontal from n at the true pole. The true pole having been found, the true equilibrium polygon was drawn and found to be some little distance above the arch axis.

It was then necessary to draw a second arch and for this second trial a three centered arch was drawn by the following method:

$$R = \frac{a^2 + h^2}{2h}$$

$$r = \frac{1}{2}\,\frac{\overline{AF}^2 + \overline{FD}^2}{FD\cos\alpha - AF\sin\alpha}$$

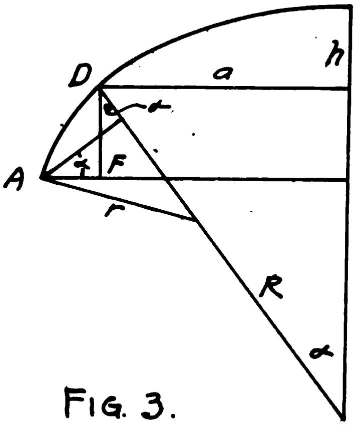

FIG. 3.

In this case a was taken as .65 x half span, and was found to be .65 x 32 = 20.8' from the crown. At this point a vertical was drawn and the curves for the arch axis and the intrados laid out as far above the equilibrium polygon as the first assumed axis was below. The extrados was made the segment of a circle. The arch ring was made the same thickness as the first trial ring except at the springing point where its depth was increased somewhat.

The dead loads and their action lines were found for this new condition and a second equilibrium polygon drawn on the right half of the arch. The second part of Table 1-a contains the values for the second trial.

The equilibrium polygon was found to coincide almost exactly with the arch axis for this trial, and the arch as laid out was adopted.

ANALYSIS OF STRESS IN ARCH RING.

The dimensions of the arch ring having been established, the maximum stresses were then determined to find if they fell within the allowable values.

The steps were as follows:

1. The arch was carefully laid out to a scale of 1"= 3 feet, Plate II, and the dimensions and radii shown.
2. The arch ring was divided into eight divisions so that $\frac{ds}{I}$ was a constant.

   (a) The length of the arch axis between the crown and the point of intersection on the axis of a line connecting the center of the extrados curve with the point where a vertical at the springing intersects the extrados, was computed.

The length of the arch axis so found was divided into ten equal parts and the radial depth at each division, of the arch ring, scaled and entered in Table 1-b.

TABLE 1-B

NUMBER OF POINT	DISTANCE FROM CROWN	RADIAL DEPTH SCALED
Crown	0	1.375
1	3.41	1.38
2	6.82	1.39
3	10.23	1.41
4	13.64	1.45
5	17.05	1.51
6	20.46	1.58
7	23.87	1.70
8	27.28	2.00
9	30.69	2.52
Springing	34.10	3.25

(b) The value of i_a was then computed and Table 2 drawn up.

i_a was determined according to Art. 178, T. & M.

I_c and I_s were determined from formulae on Page 100, T. & M.

The amount of steel reinforcement was taken as 0.7% at both the extrados and intrados and depth of steel from surface of concrete as 3"

(c) The value of $\frac{ds}{I}$ constant was computed from formula $\frac{ds}{I} = \frac{S i_a}{N}$ (Page 342, T. & M.)

$$\frac{ds}{I} = \frac{34.13 \times 2.427}{8} = 10.35$$

(d) The value of $\frac{ds}{I}$ having been found the lengths of the $\frac{ds}{I}$ divisions were determined graphically by Curve 1. The lengths of the $\frac{ds}{I}$ divisions were scaled from the curve sheet and entered in Table 3.

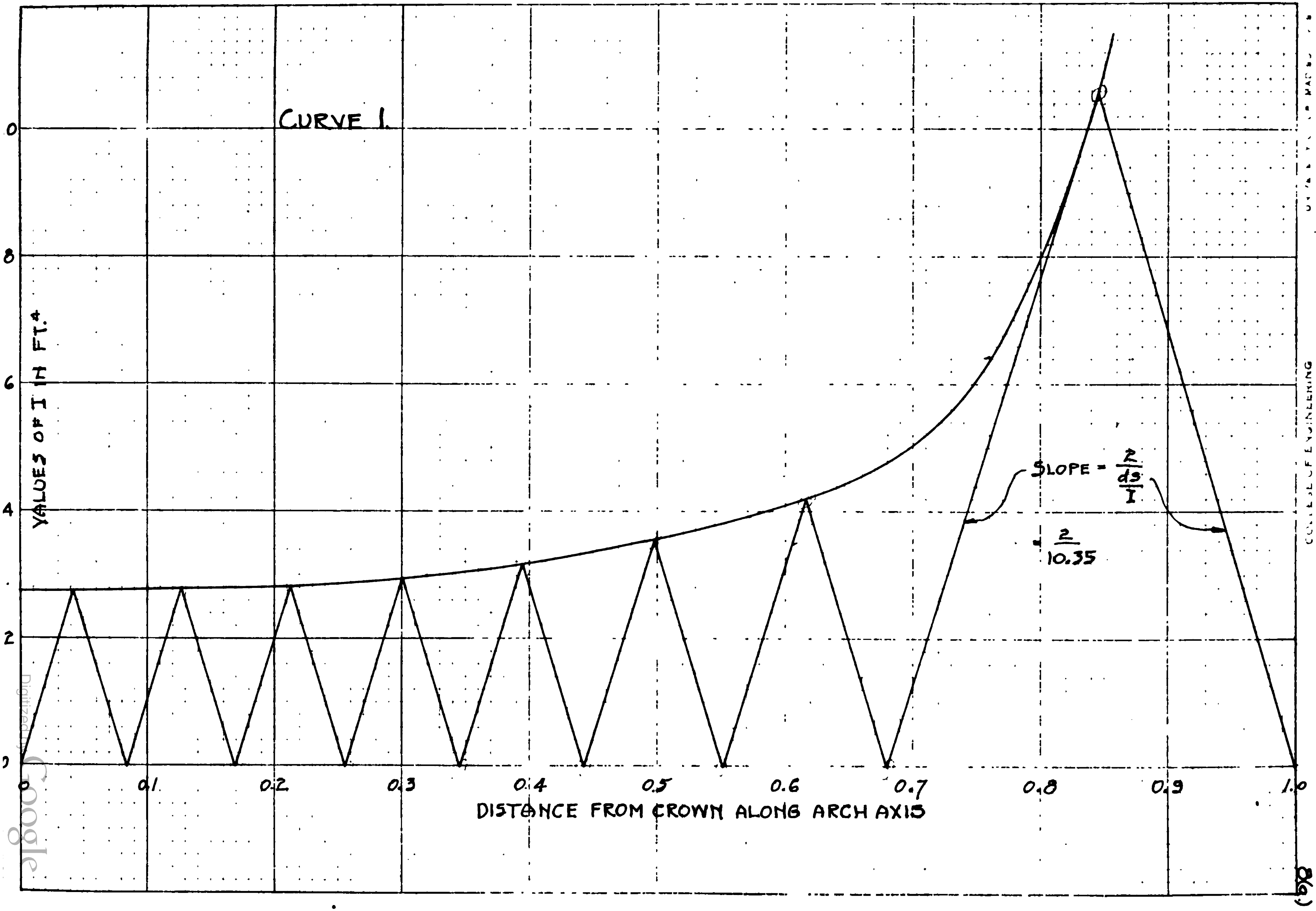

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TABLE 2.
COMPUTATIONS FOR VALUE OF i_a

POINT	RADIAL DEPTH OF CENTER "h"	h^3	$I_c = \frac{h_3}{12}$	$\frac{h}{2}$	$\frac{h}{2} - d'$	$\left(\frac{h}{2} - d'\right)^2$	$15\,I_s = 2NA\left(\frac{h}{2} - d'\right)^2$	$I = I_c + 15\,I_s$	$i_a = \frac{1}{I}$
Crown	1.375	2.6	.216	.687	.437	.190	.055	.271	3.68
1	1.38	2.63	.219	.69	.44	.193	.056	.275	3.63
2	1.39	2.68	.223	.695	.445	.198	.057	.280	3.57
3	1.41	2.80	.233	.705	.455	.207	.060	.293	3.41
4	1.45	3.05	.254	.725	.475	.226	.065	.319	3.13
5	1.51	3.44	.286	.755	.505	.255	.074	.360	2.78
6	1.58	3.94	.328	.79	.54	.291	.084	.412	2.43
7	1.70	4.91	.409	.85	.60	.36	.104	.513	1.95
8	2.00	8.00	.666	1.00	.75	.562	.162	.828	1.21
8.5	2.24	11.23	.936	1.12	.87	.756	.218	1.154	→
9	2.52	16.00	1.332	1.26	1.01	1.02	.294	1.626	.615
Spring	3.25	34.33	2.860	1.625	1.375	1.89	.545	3.405	.29

$$11\,\underline{)\,26.70}$$
$$2.427$$

$d' = 3'' = .25'$

$A = .007 \times 1 \times 1.375 = .0096$ sq. ft.

$2NA = 2 \times 15 \times .0096 = .289$

TABLE 3.
LENGTHS OF DIVISIONS TO MAKE $\frac{ds}{I}$ CONSTANT AND CENTERS OF DIVISIONS' CO-ORDINATES

NUMBER OF DIVISIONS	LENGTH	CO-ORDINATES X	CO-ORDINATES Y
1	2.88	1.44	.02
2	2.89	4.31	.20
3	2.93	7.20	.55
4	3.03	10.18	1.10
5	3.32	13.25	1.88
6	3.72	16.60	2.97
7	4.35	20.30	4.50
8	11.04	26.90	8.45
Springing	34.14	31.00	12.10

(e) The lengths of the $\frac{ds}{I}$ divisions were laid off on the arch axis, and the centers located and numbered from 1 to 8 beginning at the crown. The values of the x and y coordinates were then scaled and entered in Table 3. The crown point of the arch axis was taken as the origin of coordinates.

3. The values of the dead loads and the x coordinates of their action lines were then found and tabulated in Table 4.

TABLE 4
VALUES OF THE DEAD LOADS AND THE CO-ORDINATES OF THEIR ACTION LINES

Load Number	X Co-ordinate	Amount of Load
P_1	1.6	2090
P_2	4.8	2175
P_3	8.0	2450
P_4	11.2	2600
P_5	14.4	2960
P_6	17.6	3420
P_7	20.8	3980
P_8	24.0	4740
P_9	27.2	5850
P_{10}	30.4	7560

4. Next the arch was assumed to be cut at the crown forming two cantilevers. The cantilever moment "m" for the dead load was then computed. The value of m_L represents the moment at each of the $\frac{ds}{I}$ division center points due to the loads between the point considered and the crown, for the left half of the arch, while m_R equals the moment for the right half.
As the arch is symmetrical about the ℄ the dead load moments for each half are equal.

FOR DEAD LOAD $M_L = M_R$

A moment table was then made out (Table 5) and the values taken at the points desired, the centers of the $\frac{ds}{I}$ divisions.

TABLE 5

CANTILEVER MOMENT TABLE FOR DEAD LOAD ON ARCH

Point	Distance from Crown	Load at Point	Sum of the Loads	Distance between Points	Increment of Moment	Moment	Point
Crown	00	00	0			0	
				1.44			
1	1.44		0			0	1
				.16			
P_1	1.60	2090	2090			0	
				2.71	5660		
2	4.31		2090			5660	2
				.49	1025		
P_2	4.80	2175	4265			6085	
				2.40	10,670		
3	7.20		4265			17,355	3
				.80	3420		
P_3	8.0	2450	6715			20,775	
				2.18	14,650		
4	10.18		6715			35,425	4
				1.02	6,850		
P_4	11.20	2600	9315			42,275	
				2.05	19,100		
5	13.25		9315			61,375	5
				1.15	10,700		
P_5	14.40	2960	12,275			72,075	
				2.20	27,000		
6	16.60		12,275			99,075	6
				1.0	12,280		
P_6	17.60	3420	15,695			111,355	
				2.70	42,400		
7	20.30		15,695			153,755	7
				0.50	7850		
P_7	20.80	3980	19,675			161,600	
				3.20	63,000		
P_8	24.00	4740	24,415			224,600	
				2.90	71,000		
8	26.90		24,415			295,600	8
				0.30	7,330		
P_9	27.20	5850	30,265			302,930	
				3.20	98,000		
P_{10}	30.40	7560	37,825			400,930	
				0.60	22,700		
Spring	31.00	37,825		31.00		423,630	Spring

MOM. CHECK 11.2 × 37,825 = 424,000

5. The cantilever moments for the live load were then computed with the live load on the left half of the arch only. This load was determined on Page 2, and found to be 625 pounds per square foot. Values were tabulated in Table 6.

TABLE 6
CANTILEVER MOMENT TABLE FOR LIVE LOAD ON HALF SPAN

POINT	DISTANCE FROM CROWN	LOAD AT POINT	SUM OF LOADS	DISTANCE BETWEEN POINTS	INCREMENT OF MOMENT	MOMENT L.L. ONLY	MOMENT DL + LL
CROWN	0 0		0				
				1.44			
1	1.44		0				
				0.16			
P_1	1.60	2000	2000				
				2.71	5420		
2	4.31		2000			5,420	11,080
				0.49	980		
P_2	4.80	2000	4000			6400	
				2.40			
3	7.20		4000			16,000	33,350
				.80	3200		
P_3	8.0	2000	6000			19,200	
				2.18	13,080		
4	10.18		6000			32,280	67,700
				1.02	6120		
P_4	11.20	2000	8000			38,400	
				2.05	16400		
5	13.25		8000			54,800	116,170
				1.15	9200		
P_5	14.40	2000	10,000			64,000	
				2.20	22,000		
6	16.60		10,000			86,000	185,070
				1.00	10,000		
P_6	17.60	2000	12,000			96,000	
				2.70	32,400		
7	20.30		12,000			128,400	282,150
				0.50	6000		
P_7	20.80	2000	14,000			134,400	
				3.20	44,800		
P_8	24.00	2000	16,000			179,200	
				2.90	46,400		
8	26.90		16,000			225,600	521,200
				0.30	4800		
P_9	27.20	2000	18,000			230,400	
				3.20	57,600		
P_{10}	30.40	2000	20,000			288,000	
				.60	12,000		
SPRING	31.00	20,000				300,000	723,600

MOM. CHECK 15 × 20,000 = 300,000

6. The cantilever moments for the live load on the whole span were then computed. The arch being symmetrical the live loads were proportional and Table 7 was made out from Table 6 by multiplying the values of Table 6 by the ratio of the live loads $\frac{500}{625} = .8$

TABLE 7
CANTILEVER MOMENT TABLE - LIVE LOAD ON WHOLE SPAN

POINT	"m" - L.L on HALF SPAN	"m" - L.L. ON WHOLE SPAN	m. DL+LL ON WHOLE SPAN
1			
2	5420	4340	10,000
3	16,000	12,800	30,150
4	32,280	25,800	61,220
5	54,800	43,800	105,170
6	86,000	68,800	167,870
7	128,400	102,700	256,450
8	225,600	180,000	475,600
SPRING	300,000	240,000	663,630

7. Summation tables were made out and the values of H_o, V_o, and M_o were computed for the following conditions:
   I. Dead load only, Table 8.
   II. Live load on 1/2 span, Table 9.

In Table 8 the values of m_R and m_L are for dead load only and are equal in amount. They were taken directly from Table 5.
In Table 9 the values of m_R are zero while the values for m_L are taken from Table 6.
The values of the x and y coordinates were taken from Table 3.
The formulae for H_o, V_o, and M_o were taken from Page 338, T. & M., due attention being paid to the signs.

TABLE 8
SUMMATION TABLE

DEAD LOAD ONLY

POINT	X	y	X^2	y^2	m_R $(-)$	m_L $(-)$	$m_R + m_L$ $(-)$	$(m_R+m_L)y$	m_R-m_L	$(m_R-m_L)X$
1	1.44	.02	2.07	0	0					
2	4.31	.20	18.57	.04	5660		11,320	2260		
3	7.20	.55	51.84	.30	17,355	SAME	34,710	19,100		
4	10.18	1.10	104.00	1.21	35,425	AS	70,850	78,000		
5	13.25	1.88	176.00	3.54	61,375	m_R	122,750	231,000		
6	16.60	2.97	276.0	8.81	99,075		198,150	590,000		
7	20.30	4.50	412.	20.20	153,700		307,400	1,385,000		
8	26.90	8.45	724.	71.50	295,600		591,200	5,000,000		
Σ		19.67	1764.50	105.60	668,190		1,336,380	7,305,360		

$$H_o = \frac{N\Sigma my - \Sigma m \Sigma y}{2(\Sigma y^2 - N\Sigma y^2)} = \frac{8(-7,305,360) - (-1,336,380)\,19.67}{2 \times (19.67^2 - 8 \times 105.6)}$$

$$H_o = -32,250,000 \div (-918) = 35,100$$

$$V_o = \frac{\Sigma(M_R - M_L)X}{2\Sigma X^2} = 0$$

$$M_o = -\frac{\Sigma m + 2H_o\Sigma y}{2N} \doteq -\frac{-1,336,380 + 2(35,100)\,19.67}{2 \times 8} = -2720$$

TABLE 9
SUMMATION TABLE

LIVE LOAD ON HALF SPAN

POINT	X	y	X^2	y^2	m_R $(-)$	m_L $(-)$	m_R+m_L $(-)$	$(m_R+m_L)y$ $(-)$	m_R-m_L	(m_R-m_L) X
1	1.44	.02			0	0				0
2	4.31	.20			0	5420		1100		23,400
3	7.20	.55	SAME	SAME	0	16,000	SAME	8800	SAME	115,200
4	10.18	1.10	AS	AS	0	32,280	AS	35,500	AS	329,000
5	13.25	1.88	TABLE	TABLE	0	54,800	m_L	103,000	m_L	726,000
6	16.60	2.97	8	8	0	86,000		255,000	EXCEPT	1,430,000
7	20.30	4.50			0	128,400		578,000	+	2,610,000
8	26.90	8.45			0	225,600		1,905,000		5,840,000
Σ		19.67	1764.5	105.6		548,500		2,886,400		11,073,600

$$H_o = \frac{8(-2,886,400) - (-548,500)\,19.67}{2\left[19.67^2 - 8(105.6)\right]} = 13,400 \text{ lbs}$$

$$V_o = +\frac{11,073,600}{2(1764.5)} = +3140 \text{ lbs.}$$

$$M_o = -\frac{-548,500 + 2(13,400)\,19.67}{2 \times 8} = +1410 \text{ ft. lbs.}$$

With the values from Tables 8 and 9 the H , V , and M for the following three conditions of loading were obtained:

Case I Dead Load only.
Case II Dead Load + Live Load on half span.
Case III Dead Load + Live Load on whole span.

Case I

$$H_o = 35,100 \text{ lbs.} \quad e = \frac{-2720}{35100} = -.0775 \text{ ft.}$$
$$V_o = 0$$
$$M_o = -2720 \text{ ft.lbs.}$$

Case II

$$H_o = 35,100 + 13,400 = 48,500$$
$$V_o = 0 + 3140 = 3140 \text{ lbs.}$$
$$M_o = -2720 + 1410 = 1310$$
$$e = -1310 \div 48,500 = -.027 \text{ ft.}$$

Case III

$$H_o = 35,100 + 2(13,400)\tfrac{500}{625} = 56,500$$
$$V_o = 0$$
$$M_o = -2720 + 2(1410)\tfrac{500}{625} = -460$$
$$e = -460 \div 56,500 = -.00815 \text{ ft.}$$

8. The force polygons and equilbrium polygons were then drawn for the three conditions of loading. The true pole was found by the method given on Page 347, T. & M. The amounts and directions of the forces having been determined from the force polygon the equilibrium polygon was easily drawn.

For the first condition, dead load only, the force polygon was drawn on the vertical through the left springing and the equilibrium polygon on the left half of the arch.

For the second condition, dead load plus live load on half span, the force polygon was drawn on the vertical through the crown and the equilibrium polygon on both values of the arch.

For the third condition, dead load plus live load on the whole span, the force polygon was drawn on the vertical through the right springing, and the equilibrium polygon on the right half of the arch.

All equilibrium polygons were drawn by starting from the crown.

Plate Two shows the above work.

9. The bending moments, thrusts, shears, and eccentric distances at the centers of the eight $\frac{ds}{I}$ divisions of the arch ring were determined for the three conditions of loading. The values were entered in Tables 10, 11, and 12.

The bending moment at any point was found by formula 4, Page 339, T. & M.

The thrusts and shears were determined graphically from the force polygons, the thrust being taken as the component of the resultant pressure parallel to the arch axis, and the shear the exponent perpendicular to the axis.

TABLE 10
BENDING MOMENTS, THRUSTS AND SHEARS

H_o = 35.100 lbs
V_o = 0
M_o = −2720 ft. lbs.

DEAD LOAD ONLY

POINT	m	y	H_oy	x	V_ox	M_o	M	THRUST (lbs.)	Eccentric Distance (ft.)	SHEAR (lbs.)
SPRING	−423,630	12.10	+425,000			−2720	−1350	51,600	.026	−900
8	−295,600	8.45	+296,500			−2720	−1820	42,700	.043	+1840
7	−153,750	4.50	+158,000			−2720	+1530	38,400	.040	+800
6	−99,075	2.97	+104,200			−2720	+2405	37,200	.065	+800
5	−61,375	1.88	+66,000			−2720	+1905	36,200	.053	+700
4	−35,425	1.10	+38,600			−2720	+455	35,700	.013	+800
3	−17,355	.55	+19,300			−2720	−775	35,30	.022	+900
2	−5660	.20	+7,000			−2720	−1380	35,100	.039	+1000
1	0	.02	+700			−2720	−2020	35,100	.058	+1000
CROWN	0	0	+			−2720	−2720	35,100	.077	0

$M = m + M_o + H_oy \pm V_ox$

LEFT HALF = $+V_ox$ RIGHT HALF = $-V_ox$

TABLE 11

BENDING MOMENTS, THRUSTS AND SHEARS

H_o = 48,500 lbs.
V_o = + 3140 lbs.
M_o = − 1310 ft. lbs.

DL + LL ON LEFT HALF SPAN

	POINT	m	y	Hoy	x	Vox	Mo	M	THRUST (lbs.)	ECCEN DISTANCE (ft.)	SHEAR (lbs.)	
LEFT HALF	SPRING	− 723,600	12.10	+ 586,000	31.0	+ 97,500	−1300	− 41,400	73,000	.566	− 2800	
	8	− 521,200	8.45	+ 410,000	26.9	+ 84,500	"	− 28,000	61,300	.456	− 500	
	7	− 282,150	4.50	+ 218,000	20.3	+ 63,700	"	− 1750	54,300	.032	− 2000	
	6	− 185,070	2.97	+ 144,000	16.6	+ 52,100	"	+ 9730	52,200	.186	− 1500	
	5	− 116,170	1.88	+ 91,200	13.25	+ 41,600	"	+ 15,330	50,500	.304	− 500	
	4	− 67,700	1.10	+ 53,400	10.18	+ 32,000	"	+ 16,400	49,500	.332	+ 900	
	3	− 33,350	.55	+ 26,700	7.2	+ 22,600	"	+ 14,650	48,700	.301	+ 2000	
	2	− 11,080	.20	+ 9700	4.31	+ 13,500	"	+ 10,820	48,500	.224	+ 3500	
	1	0	.02	+ 970	1.44	+ 4,500	"	+ 4170	48,500	.086	+ 4500	
	CROWN	0	00	0	0	0	"	− 1300	48,500	.027	+ 3140	
RIGHT HALF	1	0	.02	+ 970		− 4,500	"	− 4830	48,500	.099	+1800	CHANGE SIGNS FOR USE IN TABLE 16
	2	− 5660	.20	+ 9700		− 13,500	"	− 10,760	48,600	.221	+ 900	
	3	− 17,355	.55	+ 26,700		− 22,600	"	− 14,555	49,000	.297	0	
	4	− 35,425	1.10	+ 53,400		− 32,000	"	− 15,325	49,500	.310	− 800	
	5	− 61,375	1.88	+ 91,200		− 41,600	"	− 13,075	50,000	.261	−1300	
	6	− 99,075	2.97	+ 144,000		− 52,100	"	− 8475	50,800	.167	−2200	
	7	− 153,750	4.50	+218,000		− 63,700	"	− 750	51,800	.014	−3400	
	8	− 295,600	8.45	+410,000		− 84,500	"	+ 28,600	55,200	.518	−7500	
	SPRING	− 423,630	12.10	+586,000		− 97,500	"	+ 63,570	63,000	1.01	−6800	

TABLE 12

BENDING MOMENTS, THRUSTS AND SHEARS

H_o = 56,500 lbs.
V_o = 0
M_o = − 460 ft. lbs.

DL + LL ON WHOLE SPAN

POINT	m	y	Hoy	x	Vox	Mo	M.	THRUST (lbs.)	ECCENTRIC DISTANCE (ft.)	SHEAR (lbs)	
SPRING	− 663,630	12.10	+ 684,000			−460	+ 19,910	78,000	.266	−3800	CHANGE SIGNS FOR USE IN TABLE 16
8	− 475,600	8.45	+ 477,000			"	+ 940	67,600	.014	−4500	
7	− 256,550	4.50	+ 254,000			"	− 3010	62,000	.048	−1000	
6	− 167,875	2.97	+ 168,000			"	− 335	60,100	.0056	− 700	
5	− 105,175	1.88	+ 106,000			"	+ 365	58,800	.0062	− 600	
4	− 62,225	1.10	+ 62,100			"	− 585	57,800	.0101	−600	
3	− 30,155	.55	+ 31,100			"	+ 485	57,100	.0085	−1100	
2	− 10,000	.20	+ 11,300			"	+ 840	56,700	.015	−1400	
1	0	.02	+ 1130			"	+ 670	56,500	.012	−1800	
CROWN	0	0	+ 0			"	− 460	56,500	.0081	0	

10. The thrust H_o and moment M_o at the crown due to a variation of temperature of 20°F, were computed according to formulae on Page 343, T. & M.

The bending moments, thrusts, shears, and eccentric distances at the centers of the $\frac{ds}{I}$ divisions were found and entered in Table 13.

The bending moment was computed from formula 8, Page 343, T. & M. The thrusts and shears were obtained graphically as shown on Plate Two, the thrust being the component of H_o resolved parallel to the arch axis, and the shear the component perpendicular to the axis.

$$H_o = \frac{E}{\frac{ds}{I}} \frac{ctlN}{2[N\Sigma y^2 - \Sigma y^2]}$$

$$M_o = -\frac{H_o \Sigma y}{N}$$

$E = 2{,}000{,}000$ lbs. sq. in. $= 288{,}000{,}000$ lbs. sq ft.
$\frac{ds}{I} = 10.35$
$c = .000006$
$t = 20°F \pm$
l = SPAN OF AXIS = 62'
$N = 8$ $\Sigma y = 19.67$
$2[N\Sigma y^2 - \Sigma y^2] = 918$

$$H_o = \frac{288{,}000{,}000\,(.000006)\,20\,(62)\,8}{10.35 \times 918}$$

$$H_o = +1800$$

$$M_o = -\frac{1800\,(19.67)}{8} = -4430 \text{ ft. lbs.}$$

$H_o = \pm 1800$ lbs + for a rise in temp. − for fall
$M_o = \mp 4430$ ft. lbs. − for rise in temp. + for fall

TABLE 13
BENDING MOMENTS, THRUSTS, SHEARS AND ECCENTRIC DISTANCES FOR A RISE IN TEMPERATURE OF 20° F.

$H_o = +1800$ lbs.
$M_o = -4430$ ft. lbs.
$M_t = M_o + H_o \cdot y$

POINT	y	Hoy	Mt	φ	Cos φ	TANGENTIAL COMPONENT OF Ho = THRUST	Sin φ	NORMAL COMPONENT OF Ho = SHEAR	ECCENTRIC DISTANCE
SPRING	12.10	+ 21,900	+ 17,470	46°-16'	.691	1250	.723	1300	13.97
8	8.45	+ 15,200	+ 10,770	37°-14'	.796	1430	.605	1100	7.54
7	4.50	+ 8,100	+ 3670	24°-56'	.907	1630	.422	760	2.25
6	2.97	+ 5350	+ 920	20°-03'	.939	1690	.343	620	.545
5	1.88	+ 3380	- 1050	16°-4'	.961	1730	.277	500	.607
4	1.10	+ 1980	- 2450	12°-24'	.977	1760	.215	390	1.39
3	.55	+ 980	- 3450	8°-49'	.988	1780	.153	270	1.94
2	.20	+ 360	- 4070	5°-9'	.996	1790	.090	160	2.28
1	.02	+ 36	- 4394	1°-43'	.999	1800	.030	60	2.44
CROWN	0	0	- 4430	0°-0'	1.000	1800	0	0	2.46

RESULTS FOR A FALL IN TEMPERATURE ARE OF OPPOSITE SIGN

11. The thrust and moment due to the shortening of the arch from thrust, Table 14, were computed from formulae 7 and 8, Page 343, and formula 9, Page 344, T. & M.

In formula 9 the quantity f_c is found by the formula on Page 100, T. & M., and is the average of the values found for the crown, springing, and quarter point.

The thrust due to rib shortening is negative and decreases the average compressive stress due to dead load, live load, and temperature change.

H_o in this case must be assumed and if the computed value is within 10% of the assumed value, the assumed value is used; if not, another trial made.

Two cases were considered: first, that due to dead load, live load, rise of temperature, and rib shortening; second, that due to dead load, live load, fall of temperature, and rib shortening.

CASE I

H_o AND M_o DUE TO RIB-SHORTENING FOR A RISE OF TEMPERATURE
TO FIND f_c. ASSUME $H_o = -1400$ lbs.

$$\text{Ave } f_c \text{ crown} = \frac{48{,}500 + 1800 - 1400}{1 \times 1.375 + .289} = 29{,}400\ \#^{s}/\square'$$

$$\text{Ave } f_c \text{ Pt. 5} = \frac{50{,}250 + (1800 - 1400).961}{1 \times 1.45 + .289} = 29{,}200\ \#^{s}/\square'$$

$$\text{Ave } f_c \text{ spr.} = \frac{68{,}000 + (1800 - 1400).691}{1 \times 3.25 + .289} = 19{,}300\ \#^{s}/\square'$$

$$\text{AVERAGE} = (29{,}400 + 29{,}200 + 19{,}300) \div 3 = 25{,}970\ \#/\square'$$

$$H_o = -\frac{1}{10.35} \times \frac{25{,}970 \times 62 \times 8}{918} = -1355 \text{ lbs.}$$

$$M_o = -\frac{-1355 \times 19.67}{8} = +3300 \text{ ft. lbs.}$$

CASE II

H_o AND M_o DUE TO RIB-SHORTENING FOR A FALL OF TEMPERATURE
ASSUME $H_o = -1300$ lbs.

$$\text{Ave } f_c \text{ crown} = \frac{48{,}500 - 1800 - 1300}{1.664} = 27{,}300\ \#/\square'$$

$$\text{Ave. } f_c \text{ Pt. 5} = \frac{50{,}250 + .961(-1800 - 1300)}{1.739} = 27{,}200\ \#/\square'$$

$$\text{Ave. } f_c \text{ Spr.} = \frac{68{,}000 + .691(-1800 - 1300)}{3.539} = 18{,}600\ \#/\square'$$

$$\text{AVERAGE } f_c = (27{,}300 + 27{,}200 + 18{,}600) \div 3 = 24{,}370\ \#/\square'$$

$$H_o = -\frac{1}{10.35} \times \frac{24{,}370 \times 62 \times 8}{918} = -1270 \text{ lbs.}$$

$$M_o = -\frac{-1270\,(19.67)}{8} = +3120 \text{ ft. lbs.}$$

TABLE 14
BENDING MOMENTS, THRUSTS AND SHEARS DUE TO RIB-SHORTENING

	CASE I			CASE II		
	$H_o = -1355$ lbs. $M_o = +3330$ ft. lbs. RATIO = $\frac{H_o \text{ FOR RIB-SHORTENING}}{H_o \text{ FOR TEMPERATURE}}$ = .752			$H_o = -1270$ lbs. $M_o = +3120$ ft. lbs. RATIO = $\frac{H_o \text{ FOR RIB-SHORTENING}}{H_o \text{ FOR TEMPERATURE}}$ = .705		
POINT	BENDING MOMENT	THRUST	SHEAR	BENDING MOMENT	THRUST	SHEAR
SPRING.	− 13,100	− 940	− 980	− 12,300	− 880	− 920
8	− 8,100	− 1100	− 830	− 7600	− 1010	− 775
7	− 2760	− 1230	− 570	− 2590	− 1150	− 535
6	− 690	− 1270	− 470	− 650	− 1190	− 440
5	+ 790	− 1300	− 380	+ 740	− 1220	− 350
4	+ 1840	− 1320	− 290	+ 1730	− 1240	− 275
3	+ 2600	− 1340	− 200	+ 2430	− 1250	− 190
2	+ 3060	− 1350	− 120	+ 2870	− 1260	− 115
1	+ 3300	− 1355	− 45	+ 3100	− 1270	− 42
CROWN	+ 3340	− 1355	0	+ 3120	− 1270	0

12. The bending moments and thrusts due to the different conditions of loading, rise and fall in temperature, and rib shortening, were then combined in Table 15 to determine the maximum stress conditions.

The headings of Table 15 are self-explanatory.

TABLE 15

COMBINATION OF BENDING MOMENTS AND CORRESPONDING THRUSTS WITH THE RESULTING ECCENTRIC DISTANCES

	1		2		3		4		5		6		7			8		
	D.L. ONLY		D.L + L.L. ON HALF SPAN + B.M.		D.L + L.L. ON HALF SPAN − B.M		D.L + L.L. ON WHOLE SPAN		TEMP. RISE + RIB-SHORT.		TEMP. FALL + RIB-SHORT.		TEMP. RISE + RIB-SHORTENING + COLUMN 1, 2, 3, OR 4			TEMP. FALL + RIB-SHORTENING + COLUMN 1, 2, 3, OR 4		
POINT	MOMENT	THRUST	MOMENT	THRUST	MOMENT	THRUST	MOMENT	THRUST	MOM.	THR.	MOM.	THR.	MOMENT	THRUST	ECCEN DIST.	MOMENT	THRUST	ECC. DIST.
SPRING	−1350	51,600	+63,570	63,000	−41,400	73,000	+19,910	78,000	+4370	+310	−22770	−2130	+67,940 +24,280	+63,310 +78,310		−71,170	+70,870	1.01
8	−1820	42,700	+28,600	55,200	−28,000	61,300	+940	67,600	+2670	+330	−18,370	−2440	+31,270 +3610	+55,530 +67,930		−46,370 −17,430	+58,860 +65,160	.79
7	+1530	38,400	−750	51,800	−1750	54,300	−3010	62,000	+910	+400	−6260	−2780	+2440 −2100	+38,800 +62,400		−9270	+59,220	.157
6	+2405	37,200	+9730	52,200	−8475	50,800	−335	60,100	+230	+420	−1570	−2880	+9960	+52,620	.189	−10,045 −1915	+47,920 +57,220	
5	+1905	36,200	+15,330	50,500	−13,075	50,000	+365	58,800	−260	+430	+1790	−2950	+15,070	+50,930		+17,120 +2155	+47,550 +55,850	.36
4	+455	35,700	+16,400	49,500	−15,325	49,500	−585	57,800	−610	+440	+4180	−3000	−15,935 −1195	+49,940 +58,240		+20580 +3570	+46,500 +54,800	.442
3	−775	35,300	+14,650	48,700	−14,555	49,000	+485	57,100	−850	+440	+5880	−3030	−15,405	+49,440		+20,530 +6365	+45,670 +54,070	.450
2	−1380	35,100	+10,820	48,500	−10,760	48,600	+840	56,700	−1010	+440	+6940	−3050	−11,770	+49,040		+17,760	+45,450	.392
1	−2020	35,100	+4170	48,500	−4830	48,500	+670	56,500	−1090	+445	+7490	−3070	−5920 −415	+48,945 +56,945		+11,660	+45,430	.257
CROWN	−2720	35,100	—	—	−1300	48,500	−460	56,500	−1090	+445	+7550	−3070	−2390 −1550	+48,495 +56,945		+7090	+53,430	.133

13. The shears were next tabulated in Table 16. The maximum positive shears are found by adding to column 5 the greatest positive shear in columns 1 to 4.

The maximum negative shears were found by adding to column 6 the greatest negative value in columns 1 to 4.

The shearing stresses were found to be small.

TABLE 16
COMBINATIONS OF SHEARS AND RESULTING UNIT VALUES

	1	2	3	4	5	6	7	8	9	10	11
POINT	D.L. ONLY	D.L.+LL. ON HALF SPAN		D.L.+ L.L ON WHOLE SPAN	CASE I TEMP. RISE + RIB-SHORT	CASE II TEMP. FALL + RIB-SHORT.	MAXIMUM SHEAR		CROSS SECTION SQ. FT.	MAX. UNIT SHEAR LBS/SQ.FT	MAX UNIT SHEAR LBS/SQ. IN.
		+ SHEAR	− SHEAR				+	—			
SPRING	− 900	+ 6800	− 2800	+ 3800	+ 320	− 2220	+ 7120	− 5020	3.25	2190	15.2
8	+ 1840	+ 7500	− 500	+ 4500	+ 270	− 1875	+ 7770	− 2375	2.20	3530	24.5
7	+ 800	+ 3400	− 2000	+ 1000	+ 190	− 1295	+ 3590	− 3295	1.60	2240	15.6
6	+ 800	+ 2200	− 1500	+ 700	+ 150	− 1060	+ 2350	− 2560	1.50	1710	11.9
5	+ 700	+ 1300	− 500	+ 600	+ 120	− 850	+ 1420	− 1350	1.46	975	6.8
4	+ 800	+ 900	+ 800	+ 600	+ 100	− 665	+ 1000	+ 135	1.42	705	4.9
3	+ 900	+ 2000	0	+ 1100	+ 70	− 460	+ 2070	− 460	1.40	1480	10.3
2	+ 1000	+ 3500	− 900	+ 1400	+ 40	− 275	+ 3540	− 1175	1.39	2550	17.7
1	+ 1000	+ 4500	− 1800	+ 1800	+ 15	− 100	+ 4515	− 1900	1.38	3280	22.8
CROWN	0	+ 3140	—	0	0	0	+ 3140	0	1.375	2280	15.8

14. The maximum stresses due to the bending moments and eccentric distances in Table 15 were computed. The formulae on Pages 98 to 100, T. & M., were used to determine the maximum values. The table on Page 98 was used to determine the case under which the section fell. The quantities p and $\frac{e}{h}$ were known and from them and charts on Pages 287 and 288 the values of M/bh^2f_c were determined.

Formula 13 on Page 104, T. & M., was used to determine the tension in the steel, the value of k in the formula having been found from the diagram on Page 106, T. & M.

TABLE 17
MAXIMUM STRESSES

A = .0096 Sq. Ft.
d' = 3" = .25'

								Maximum Compression in Concrete Case I or Case II					Maximum Tension in Steel Case II						
Pt.	h	$p = \frac{A}{bh}$	h^2	e	$\frac{e}{h}$		$\frac{M}{bh^2f_c}$	M	$\frac{M}{bh^2}$	f_c lbs/sq ft	f_c lbs/sq in	Where & When Occurs	k Case II	$d = h - d'$	kh	$\frac{d}{kh}$	$\frac{d}{kh} - 1$	nf_c	f_s = lbs/sq in
SR	3.25	.003	10.55	1.01	.311	II	.114	-71,700	6800	59,600	414	I Fall	.69	3.0	2.24	1.34	.34	6210	2110
8	2.20	.0043	4.84	.79	.359	II	.123	-46,370	9580	77,800	540	I Fall	.637	1.95	1.40	1.39	.39	8100	3160
7	1.60	.006	2.56	.157	.098	I	.076	-9270	3620	47,600	331	I Fall							
6	1.50	.0064	2.25	.189	.126	I	.088	+9960	4430	50,400	350	E Rise							
5	1.46	.0066	2.13	.36	.246	II	.130	+17,120	8050	62,000	430	E Fall	.87	1.21	1.27	.954	—		
4	1.42	.0067	2.02	.442	.311	II	.135	+20,580	10,200	75,500	525	E Fall	.75	1.17	1.065	1.10	.10	7880	788
3	1.40	.0068	1.96	.450	.322	II	.137	+20,530	10,470	76,400	530	E Fall	.732	1.15	1.025	1.123	.123	7950	980
2	1.39	.0069	1.93	.392	.282	II	.134	+17,760	9200	69,100	480	E Fall	.80	1.14	1.11	1.03	.03	7200	216
1	1.38	.0069	1.90	.257	.186	I	.107	+11,660	6140	57,400	398	E Fall							
CR	1.375	.007	1.89	.133	.097	I	.078	+7090	3750	48,100	332	E Fall							

15. Abutment Design.

After the arch ring had been analysed, the design of the abutment was taken up.
Three conditions of loading were considered, as follows:

1. D.L.+ L.L. on half span opposite abutment.
2. D.L.+ L.L. on half span adjacent to abutment with L.L. over abutment.
3. D.L.+ L.L. on whole span with L.L. over abutment.

Methods described in Volumn III of Hool's Reinforced Concrete Construction, Pages 54 to 57, were used as a guide.

Load Conditions.

Bearing power of soil = 5.5 tons per square foot.
Weight of concrete - 150 pounds per cubic foot.
Weight of earth fill - 100 pounds per cubic foot.
Ties and ballast - 200 pounds per lineal foot.
Live load - 500 pounds per lineal foot.

The dimensions of the abutment, as well as the resultant thrusts, are shown on Plate Two.

Condition 1 was found to be the one giving the maximum soil pressure.

From Plate Two the vertical component of the thrust on the base was found graphically to be 102,000 pounds, and the eccentric distance 2.5 feet.

Then soil pressure at the heel of abutment was determined from formula

$$P = \frac{R_v}{b} + \frac{6 R_v e}{b^2}$$

$$P = \frac{102{,}000}{18} + \frac{6\,(102{,}000)\,2.5}{18^2} = 10{,}360^{\#} = 5.18 \text{ Tons/sq. ft.}$$

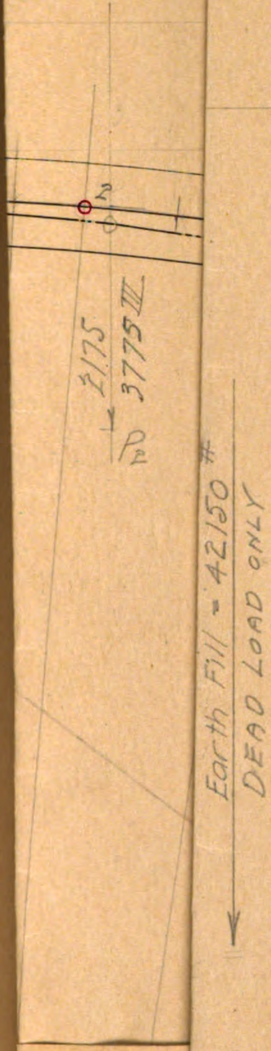
All of the thrusts were found to strike within the middle third of the base.

Approved, June 17, 1920.

W S Kinne

Associate Professor
Structural Engineering
University of Wisconsin

PLATE 2.



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